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SPECIAL PROCEDURES FOR PAVEMENT DESIGN

By L. A. Palmer

SOIL MECHANICS AND FOUNDATIONS DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

SPECIAL PROCEDURES FOR PAVEMENT DESIGN

By L. A. Palmer[1]

Synopsis

In building airfield pavements to resist the heavy loads and thermal shock produced by improved types of aircraft, the Bureau of Yards and Docks, United States Department of the Navy, has established the testing procedures and design criteria described herein.

Material must fulfil special requirements to be used on runway ends, and the structural properties of the base course and subbase for the entire runway must conform to high standards. This paper describes the requirements of such material, the types of tests used, and the principles on which the pavement construction methods are based.

Introduction

As a consequence of the development of new and heavier types of naval aircraft, the Bureau of Yards and Docks of the United States Department the Navy has established tentative testing procedures and general design criteria for the construction of new pavements and the reinforcement of existing ones.

Simultaneously with the pursuance of its huge pavement construction program, the Navy has planned and has conducted research in its laboratories in an effort to find new materials and procedures for constructing pavements having high resistance to the repeated thermal shock produced by the hot exhaust gases from jet aircraft. Such materials, if found, should have the following properties: They must be economical; they must have low thermal conductivity so that they may be used as a cover to protect existing pavements; they must possess structural stability; and they must have low thermal expansivity. Also, they must resist the solvent action of petroleum distillates that are used as fuels. These properties are required particularly in the end zones of runways, at junctures of taxiways and runways, in warm-up aprons where

Note.—Written comments are invited for publication; the last discussion should be submitted by November 1, 1953.

[1] Eng. Consultant, Soil Mechanics and Paving, Bureau of Yards and Docks, U. S. Dept. of the Navy, Washington, D. C.

jet planes check power before taking off, and to a somewhat lesser extent in taxiways where the jet aircraft must stop and start frequently. In the intermediate stretches of runways between the end zones, no special materials are needed; but the design of bituminous mixes for surfacing, and of the base course and subbase for flexible pavements, must be adequate for single tire loads of 20,000 lb with a tire inflation pressure of 250 lb per sq in. and for single tire loads as great as 50,000 lb with a tire pressure of 150 lb per sq in.

The complete stress analysis for multi-layered systems of infinite lateral extent and of variable (yet finite) thicknesses of the separate-layered components remains to be accomplished. Donald M. Burmister,[2] M. ASCE, has presented a precise solution limited for the most part to two-layered systems. His published solution applies only to the settlement or deflection of points on the axis of symmetry when a load is distributed uniformly over a circular area (either by means of a rigid plate or by other means). Expressions for shearing stresses and normal stresses within and below the separate-layered components apparently await further studies.

For the case of a load that is uniformly distributed over a circular area on a single homogeneous layer of great depth, both S. D. Carrothers[3] and A. E. H. Love[4] have shown that a basin-shaped surface extends from the perimeter to a depth that depends on the assumed value for the Poisson ratio, at all points of which surface the shearing stress is a maximum and equal to w/π, w being the unit load. For a Poisson ratio of 0.5, the depth of maximum shear is about 0.7 r, in which r is the radius of the uniformly loaded area. In the case of multi-layered systems, such as flexible pavements, the depth to which the occurrence of maximum shearing stresses extends is not known. Because all the layered components of a flexible pavement tend to spread the load more than does the subgrade, certainly the region of maximum shear does not extend to depths greater than 0.7 r in flexible pavements, and an assumption on the side of safety is that shearing stresses equal to w/π extend to this depth. For a tire load having an inflation pressure of 250 lb per sq in., the actual contact pressure between the tire and the pavement is 250 lb per sq in. plus 10% of this amount, or 265 lb per sq in. Thus, the maximum shearing stress (assuming the tire footprint to be circular) is 84.4 lb per sq in. For the greater tire load of 50 kips, with tire pressure equal to 150 lb per sq in., the depth within which the shearing stress is a maximum is approximately 7 in.

Thus, approximately 7 in. of the flexible-type pavement must have shearing resistance equal to not less than $165/\pi = 52.5$ lb per sq in. This stress condition demands an especially tough surfacing and a base course, the top few inches of which must have high shearing resistance. For the tire load of 20 kips at a tire pressure of 250 lb per sq. in., the shearing resistance must be not less than 84.4 lb per sq in. in the upper 5 in. of pavement. This stress condition demands a surfacing of unusual toughness.

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[2] "The Theory of Stresses and Displacements in Layered Systems and Application to the Design of Airport Runways," by Donald M. Burmister, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., Vol. 23, 1943, pp. 126–148.

[3] "Elastic Equivalence of Statically Equipollent Loads," by S. D. Carrothers, *Proceedings*, International Mathematical Cong., Vol. 2, Toronto University Press, Toronto, Ont., Canada, 1924, p. 527.

[4] "The Stress Produced in a Semi-Infinite Solid by Pressure on Part of the Boundary," by A. E. H. Love, *Philosophical Transactions*, Royal Soc. of London, Series A., Vol. 228, 1929.

Experience has evidenced that the required total or over-all flexible pavement thickness is that which will assure a settlement not exceeding 0.2 in. under the severest conditions of loading—which must take into account both tire load and tire pressure. Thus, it may be shown that for the two conditions—a 20-kip tire load with tire pressure equal to 250 lb per sq in., and a 50-kip tire load with tire pressure equal to 150 lb per sq in.—the latter condition is the determining factor in estimating the required total thickness of flexible pavement.

Therefore, the high tire pressures that characterize new models of naval aircraft require an especially tough, asphaltic concrete surfacing and a base course of which at least the top few inches must have a correspondingly high shearing resistance. Experience has demonstrated the economic feasibility of designing and constructing asphalt-surfaced pavements that meet these requirements. The total pavement thickness required for the 50-kip tire load at a tire pressure or 150 lb. per sq in. is determined by loadings on trial pavement sections having a base course of fixed thickness and a subbase of variable thickness. A circular steel plate, 20 in. in diameter, is used for this purpose. The required pavement thickness is obtained by first loading the compacted subgrade and computing the modulus of compression E_2 of the subgrade, then obtaining E_1 for the pavement by loading the base course after this course has been compacted on the subgrade and thereafter using Mr. Burmister's influence curves.

For the design of concrete pavement, loading tests are made either on the compacted subgrade or on a prepared subbase of variable thickness to determine the modulus, k, for use in the Westergaard analysis.

Essential Features of Pavement Design

For the flexible-type pavement, quality (especially toughness) of the pavement is a consideration equal in importance to pavement thickness. The required pavement quality is determined by the tire pressure, and the required over-all pavement thickness depends on the magnitude of the greatest anticipated wheel load, with proper consideration being given to tire pressure and the essential differences in requirements for loads on single tires, dual tires, and dual-tandem tires. On the basis of experience, field tests, and observations, quality indexes have been tentatively adopted by the Bureau of Yards and Docks for the following paving components and procedures: Asphaltic concrete surfacing, base courses for flexible pavements, subbases, subgrade preparation, the use of trial pavement sections, and proving of the design by studying pavement performance.

Asphaltic Concrete Surfacing.—The design of the bituminous mix for both the binder course and the wearing course is based on the Marshall test. Traffic tests made for the Bureau of Yards and Docks by the Corps of Engineers, United States Department of the Army, at the Waterways Experiment Station, at Vicksburg, Miss.,[5] in addition to the Marshall stability tests made with cored specimens of asphaltic concrete taken from pavements in use at naval

[5] "Effect of Traffic with Small High-Pressure Tires on Asphalt Pavements," *Technical Memorandum No. 3-314*, Corps of Engrs., U. S. Dept. of the Army, Vicksburg, Miss., June, 1950.

air stations, have led to the adoption of the following standard requirements for the binder course:

Test Property	Values
Marshall stability (pounds)	$> 1{,}500$
Flow (hundredths of an inch)	< 16
Voids (%)	$4 < n < 6$
Voids filled with asphalt (%)	$70 < X < 82$

The standard requirements for the wearing course are the same with the following exceptions: Voids (%) $3 < n < 5$; and voids filled with asphalt (%) $75 < X < 85$. Asphalt paving mixtures are designed for a tire pressure of 250 lb per sq in., based on tests of cylindrical specimens compacted in a mold by applying seventy-five blows on each flat surface with the commonly used Marshall equipment. The optimum asphalt content is the average of the different asphalt contents at the following points on curves (test property as related to asphalt content):

Test property	Point for selection of asphalt content
Stability	Peak of curve
Unit weight, total mix	Peak of curve
Unit weight, aggregate only	Peak of curve
Voids (%)	5
Voids filled with asphalt (%)	78

Specifications for the placing and rolling of binder and wearing courses require that compaction produce a density that is 98% of the density (corresponding to the optimum asphalt content) obtained in the laboratory compaction of the specimen, which requires seventy-five blows per flat surface.

A characteristic of mixes designed in this manner is the tendency toward leanness in asphalt—that is, "dryness." One of the greatest sources of danger when using high-pressure tires is the extraction of asphalt from the surfacing material as the result of kneading action of the tires, best described as "working" of the asphalt out of the surfacing material. A certain residual void space to contain asphalt "worked out" by progressive compaction under unlimited traffic and a low asphalt content to permit desirable aggregate interlocking and maximum internal frictional resistance are absolutely necessary. Dense gradation of hard and durable aggregates and a careful determination of optimum asphalt and mineral filler content are sufficient for meeting all requirements. The asphalt penetration usually ranges from 70 to 100.

New asphalt surfacing construction is proof-tested by repeated coverages (cycles of operation) over tracks of two-tire widths with vehicles of the type shown in Fig. 1. A thousand coverages are sufficient for a test. The tire inflation pressure for these tests is 250 lb per sq in., and the load on a single tire is 9,000 lb. The Marshall stability and flow, the percentage of voids, the percentage of voids filled with asphalt, and the bulk density are determined both before and subsequent to the traffic tests from cored specimens, 4 in. in diameter.

The compacted asphalt surfacing (binder course plus wearing courses) is 3 in. thick for the types of tire loads described, and the seal coat usually is omitted at the time of construction.

Base Courses for Flexible Pavements.—The following types of base courses are being constructed mechanically: Stabilized aggregate, dry-bound macadam, water-bound macadam, limerock, bituminous concrete (black base), and penetration macadam. For overlays on existing flexible pavements and existing concrete pavements the black base type and the asphalt penetration macadam type usually are preferred. After tack coating these adhere well to the old pavement. For runway extensions and other new pavement construction, mechanically stabilized aggregate tends to be the most economical generally. Dry-bound macadam and water-bound macadam permit slightly thinner pavements than does stabilized aggregate, and they are often preferred where there are long hauls of aggregates or other transportation difficulties. The use of stabilized aggregate and limerock effects a substantial saving in the time required for construction of the base.

A minimum soaked (4 days of soaking) California Bearing Ratio (CBR) of 80 is recommended as a quality index for the mechanically stabilized or select aggregate base course. This index is made necessary by the high shearing stresses caused by high-pressure tires. In nearly all instances, this minimum has been exceeded without difficulty. Of course, it is not practicable to utilize the CBR test in the design of the other types of bases. For the black base, a minimum Marshall stability of 1,000 lb and a flow value not exceeding 16 is the usual requirement.

The only quantitative index to the relative bearing values of in-place base courses, such as the dry-bound and water-bound macadams, the black base, and the penetration macadam, is the plate loading test. This index is expressed in terms of a "modulus," E_1, obtained after first loading the prepared subgrade with a steel plate 20 in. or 30 in. in diameter, and thereafter loading with the same plate a layer of known thickness of the base course constructed on the same prepared subgrade of measured modulus, E_2. The procedure for computing both E_1 and E_2 with the aid of influence curves has been completely described by Mr. Burmister.[2] The measured moduli of base courses tend to vary over a wide range, those for stabilized aggregate base course being generally between 25,000 lb per sq in. and 100,000 lb per sq in. The moduli for well-constructed penetration macadam, black base, and water-bound macadam usually tend to exceed 100,000 lb per sq in., and, in some instances, are as high as 200,000 lb per sq in. There has been little work done toward a careful and thorough analysis of the large number of load-test data obtained (1953), but it is apparent that such an analysis will definitely serve to classify base courses from the standpoint of their load-distributing characteristics. This classification is particularly desirable in the case of bituminous-treated aggregates and other processed aggregates for which the soaked CBR test has no application. The importance of quality of the flexible-type pavement has been minimized by the overemphasis attached to pavement thickness alone. The attainment of high quality of pavement with respect to high shearing resistance and load-

distributing properties is a matter of both good design and good construction. Neither one without the other can produce an adequate pavement.

Fig. 2 shows the general relation or lack of relation between soaked CBR-values and the moduli, E_2 for subgrades and E_1 for base courses. The moduli were obtained without presoaking any of the materials. This difference should be of relatively little importance in the case of base courses consisting of densely graded select aggregates of the 2-in. size and smaller. A lack of relationship would be expected in the case of silt and clay subgrades. In this case, the soaked CBR-values and not the in-place CBR-values were measured, whereas the plate loading tests were made either on the subgrades under existing pavements or on compacted subgrades that were at or near their optimum moisture contents during the course of construction of trial pavement sections. Never-

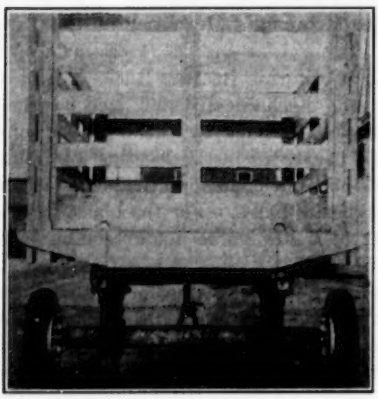


FIG. 1.—LOADING VEHICLE FOR NEW ASPHALT SURFACING

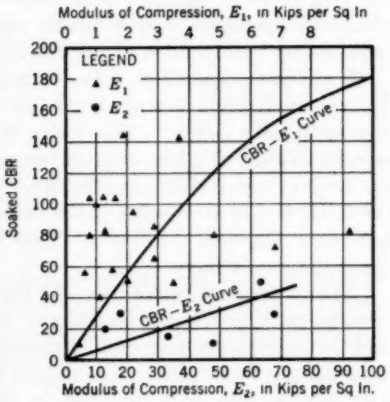


FIG. 2.—RELATION BETWEEN MODULI OF COMPRESSION AND SOAKED CBR-VALUES

theless, a correlation between CBR-values and E_2-values of subgrades is indicated in Fig. 2.

Subbases.—The general policy for the design of subbases is to construct progressively tougher pavement components from the subgrade up to and including the surfacing for flexible pavements, with the base course designed to function as the principal structural component. Contingent on relative costs and availabilities of materials, the general policy is to maintain also a fixed thickness of base course in constructing trial pavement sections and to vary the pavement thickness in these sections by varying that of the compacted subbase. The fixed thickness of the base course is intended to insure against failure by shear within the pavement caused by excessive tire pressures. The thickness of the subbase is then estimated from the plate loading test as being that thickness required to avoid the subgrade yielding beyond an extent such that the maximum anticipated tire load causes a plate settlement exceeding 0.2 in. In a few instances, the cost of subbase materials that were not near the site has been equal to or greater than the cost of good base course material

that was locally available. In all such cases the subbase is omitted, and the thickness of the base course is varied in the trial sections.

For flexible pavements, suitable subbase materials commonly used are chert, caliche, pit-run uncrushed gravel and sand, decomposed or partly decomposed granite, sand-clay, and mixtures of sand-clay with shell or crusher-run stone.

The essential, recommended requirements for subbase material for flexible pavements are as follows: A 4-day soaked CBR of not less than 45; a liquid limit of material passing the No. 40 sieve of not more than 25%; and a plasticity index less than 8%. The fraction passing the No. 200 sieve shall not exceed one half the fraction passing the No. 40 sieve. For concrete pavements, the recommended minimum soaked CBR is 30 and all other requirements are the same as those for subbase in flexible-type pavements. Base courses, comparable to those in flexible pavements, are not used to support concrete pavements. The subbase for concrete pavements must be dense, relatively impervious, and of smooth finish. To be avoided are open-type subbases and pervious-type subbases, which collect and hold water and into which the concrete, when poured, can penetrate and bond.

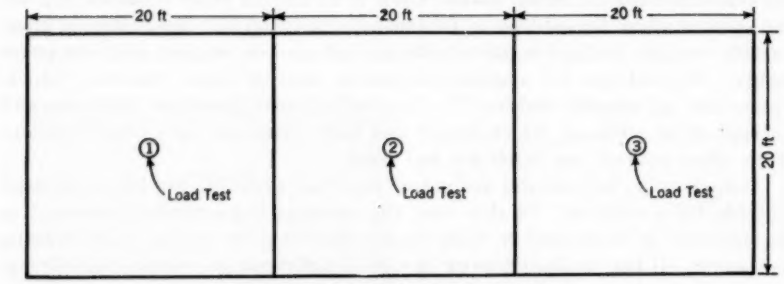
Occasionally, the natural ground or material available for fills is in itself suitable for a subbase. In this case, the material is loosened or removed as required and is compacted to those depths indicated by loading tests as being necessary. If the in-place density in cuts is sufficient, no removal, scarifying, or compaction is required.

Subgrade Preparation.—To provide adequate drainage of the site during the period of construction, outfall ditches are dug in the subgrade, and transverse grades are maintained on the subgrade to permit runoff. Failure to do this is the usual cause of soft and spongy subgrades that render impossible the effective compaction of the subbase. Where there is to be fill over a mud layer that will consolidate under the dead load produced by the fill plus the pavement, the mud must either be removed and replaced with more granular or stable material or be stabilized. Stabilization is achieved by constructing interconnecting vertical sand drains and overloading the mud by adding extra surcharge which is cut away and removed after a few weeks when the mud becomes so well consolidated that no slow settlement will be realized with final grades established. Because the time-consolidation relation for very peaty soil differs radically from that of normal silt and clay soils, no attempt is made to stabilize peat by the use of sand drains. Peat soils must be removed and replaced.

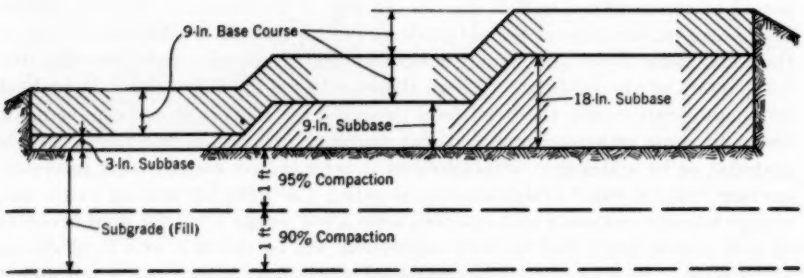
Trial Pavement Sections.—Much could be written about trial pavement sections. The proper selection of sites for trial sections within the area to be paved requires elaborate description and nothing can be substituted for experience. The trial section is a full-size model test. A loading test on a trial section is a test of the finished product—the pavement—and it reflects the combined influence of a multiplicity of factors, each unpredictable from "piecemeal" studies in the laboratory. Among other things, the construction of trial sections tends often to disclose unexpected difficulties in compacting the pavement components. It is well to discover such difficulties prior to preparing the final plans and specifications. As an example—it was found in one instance to be inadvisable to construct a penetration macadam base course on a sand-clay

subbase. In the trial section the sand-clay, compacted to more than 95% of maximum density (by the laboratory compaction procedure given by the modified (Corps of Engineers) form of Test T99-49 of the American Association of State Highway Officials[6]), was readily penetrated when the coarse aggregate was rolled on it. Changing the base course to a select, graded, and mechanically stabilized type was more feasible than changing the subbase and, when this alteration was made, all went well and issuance of a change order after award of the contract was avoided.

By loading first the compacted subgrade and then a subbase and base course of arbitrary thicknesses to find the separate moduli of these materials, an esti-



(a) PLAN OF TRIAL PAVEMENT PANELS



(b) CROSS SECTION

FIG. 3.—TYPICAL SECTION OF A TRIAL PAVEMENT

mate of the required pavement thickness may be made by using the Burmister procedure and influence curves. A trial section comprised of three adjoining plots not less than 20 ft by 20 ft is then constructed so that the combined thickness of the base course and the subbase is equal to the estimated value in one plot, one half to two thirds of the estimated thickness in a second plot, and 1.5 times the estimated required thickness in a third plot (see Fig. 3). Load tests then are made on each plot with a circular steel plate of area equal to the tire footprint of the greatest anticipated tire loading that the pavement is to accommodate. Corrections of the observed plate deflections under the full

[6] "Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II," adopted by the Am. Assn. of State Highway Officials, Washington, D. C., 1950.

load in each case may be made for any assumed future increase in moisture content of the subgrade that may seem reasonable. If the subgrade modulus, E_2, is known for various subgrade moisture contents, it is preferable to make the correction by using theory. The design basis for flexible pavement thickness is an upper limit of settlement of 0.2 in. For a rigid plate of radius, r,

$$E_2 = \frac{3\,\pi\,w\,r}{8\,\Delta} \qquad (1)$$

in which w is the average pressure of the plate on the soil and Δ is the settlement. Succeeding paragraphs demonstrate the design procedure.

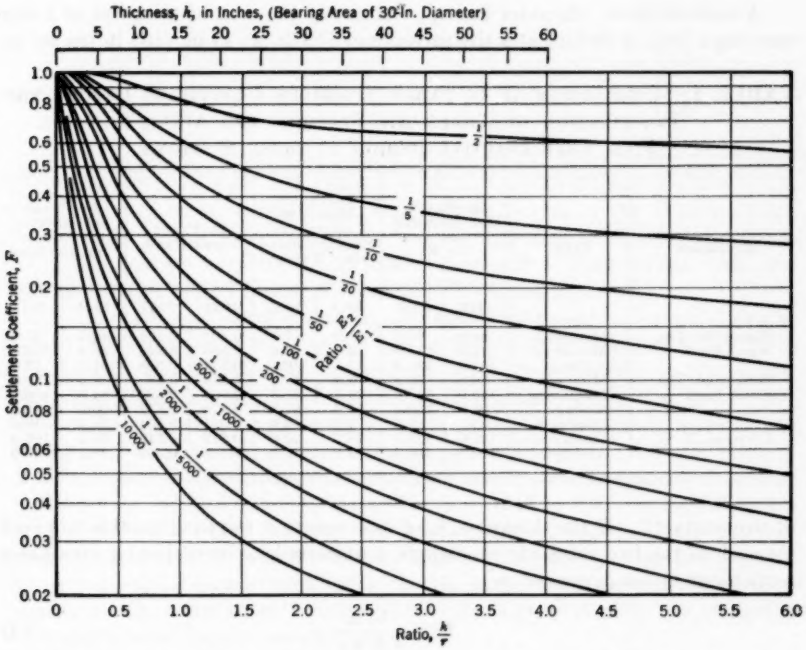


FIG. 4.—INFLUENCE CURVES FOR THE SETTLEMENT COEFFICIENT, F, FOR THE TWO-LAYER SYSTEM

By loading on a subgrade compacted to 96% of maximum density at optimum moisture, with a 30-in.-diameter loading plate, the observed value for w at $\Delta = 0.2$ in. was 40 lb per sq in. Then $E_2 = 3{,}535$ lb per sq in. by Eq. 1. When the same subgrade was loaded four years later, using the same plate, the subgrade moisture at the same location was found to have increased to 4% above optimum and the observed p-value for this condition was 32 lb per sq in. The corrected value of E_2 became 2,830 lb per sq in.

The Burmister settlement factor is

$$F = \frac{E_2\,\Delta}{1.18\,w\,r} \qquad (2)$$

By loading on 12 in. of base course for the first condition, the observed w-value was 130 lb per sq in. at a Δ-value of 0.2. This yields an F-value equal to 0.307. If the thickness of the reinforcing or pavement layer is h, and is equal to 12 in. (the equivalent of 0.8 r), a set of influence curves developed by Mr. Burmister (see Fig. 4) yields a value of $\frac{1}{80}$ for the ratio, $\frac{E_2}{E_1}$ corresponding to the F-value for this case. Therefore, $E_1 = 282{,}900$ lb per sq in. Similarly, for the second (wetter subgrade) condition, and with E_1 already evaluated, $\frac{E_2}{E_1}$ is approximately $\frac{1}{100}$.

A circle of 20-in. diameter has approximately the area of the print of a tire carrying a load of 50 kips at a tire pressure of 150 lb per sq in. (165 lb per sq in.

TABLE 1.—Comparison of In-Place Subgrade Conditions Before the Inactivation of Three Air Stations and After a Five-Year Inactive Period at these Stations

Air station	Value	In-Place Dry Density, in Lb. per Cu Ft		In-Place Moisture Content in Percentage of Dry Weight		In-Place Voids Ratio		In-Place Saturation, in Percentages	
		1944	1951	1944	1951	1944	1951	1944	1951
Kingsville, Tex. North Field	Minimum	94.0	91.0	7.0	7.1	0.501	0.460	44.3	33.0
	Maximum	112.0	115.0	18.0	28.0	0.865	0.844	80.9	100.0
	Average	101.4	105.0	12.1	16.5	0.714	0.598	61.6	73.0
Kingsville, Tex. South Field	Minimum	80.6	89.0	9.4	11.0	0.554	0.435	36.5	61.0
	Maximum	108.0	117.0	24.1	32.0	0.907	0.886	84.0	100.0
	Average	94.6	100.4	17.3	21.1	0.774	0.672	60.7	84.5
Edenton, N. C.	Minimum	92.0	97.8	6.0	4.9	0.391	0.354	26.5	19.6
	Maximum	127.0	123.5	21.3	20.0	0.817	0.710	99.1	100.0
	Average	110.4	105.5	14.6	10.6	0.514	0.585	76.1	48.6

of tire contact). If the thickness, h, of base course is required for this tire load for each of the two subgrade conditions, a uniform load on a circular area has a settlement factor expressed as

$$F = \frac{E_2 \Delta}{1.5\, w\, r} \qquad (3)$$

For the subgrade at optimum moisture, $F = 0.286$. Interpolation in Fig. 4, using the proportion, $\frac{E_2}{E_1} = \frac{1}{80}$, yields $\frac{h}{r} = \frac{h}{10} = 0.87$, or $h = 8.7$ in.

For the wet subgrade condition, $F = 0.229$ and $\frac{E_2}{E_1} = \frac{1}{100}$, as found. Interpolating again from Fig. 4, $h = 10.6$ in. Thus, the 12 in. of base course (exclusive of 2.5 in. of tough asphaltic concrete surfacing) was more than ample for the design load, even for the wettest condition of subgrade found anywhere under the pavement in an evaluation study of the field.

As an aid in the substitution of facts for prevalent suppositions concerning the probable wetting of subgrades under airport pavements, the data of Table 1

are shown. Subgrade conditions prior to and following a long period of inactivity of three airfields are shown. During the period of inactivity (about five years in each case) there was neither plane traffic nor pavement maintenance. The subgrades of the two airfields at Kingsville, Tex., are clays of the CL and CH plasticity types and the subgrade at the field at Edenton, N. C., varies from SF to ML. The surfacings at all three fields were badly cracked, and every condition favored the wetting of the subgrade. Normally, subgrades under airport pavements in use do not show these extreme conditions.

The average optimum moisture contents of the subgrades at the three air stations were found to be the following:

Air station	Optimum moisture (%)
North Field, Kingsville	12.5
South Field, Kingsville	16.1
Edenton	12.8

In 1951, following the long period of inactivity, the average percentage increases above optimum (see Table 1) were 4.0 and 5.0 for the first two of the aforementioned fields, respectively, whereas in the case of the Edenton field, the subgrade moisture dropped to 2.2% below optimum. Significantly, however, the in-place subgrade density also increased simultaneously with the increase of subgrade moisture at the Kingsville airfields.

The average maximum subgrade densities were found to be 117.6 lb per cu ft, 112.2 lb per cu ft, and 119.2 lb per cu ft for North Field and South Field at Kingsville and at Edenton, respectively. For 1951 conditions, the corresponding in-place dry densities were 89.3%, 89.5% and 88.5% of the maximum densities, respectively, representing a small increase in percentage of maximum density for the Kingsville stations and a small decrease for the Edenton station.

The data of Table 1 may be summarized still further as follows:

Kingsville, North Field.—In the period from 1944 to 1951, during five years of which (1946 to 1951) the field was inactive, the average in-place subgrade density increased 3.6 lb per cu ft, the average in-place moisture increased 4.4%, the average voids ratio diminished by 0.116, and a corresponding higher percentage of the voids space became filled with water; that is, the subgrade was consolidated slightly during this period.

Kingsville, South Field.—The average in-place subgrade density increased by 5.8 lb per cu ft and the average in-place moisture increased 3.8%. Again, the average voids ratio decreased (that is, the subgrade consolidated slightly) with a consequent increase in percentage saturation.

Edenton.—In this case, the average in-place density decreased by 4.9 lb per cu ft, but the average in-place moisture content also decreased by 4.0%. The average voids ratio increased with a consequent decrease in the average percentage saturation. In other words, the subgrade rebounded or recovered in a manner similar to that observed when there is a release of the load on a loaded soil mass.

The asphalt surfacing at the three airfields had no seal coat application during the period of inactivation and tended to become brittle, to oxidize, and

to deteriorate during the five years they were without traffic. Even with these unusually favorable conditions for wetting subgrade, there seems to be no reason for extreme conservatism in applying a correction factor to the design thickness of a flexible pavement as indicated from loading tests on trial sections. At a total of seventy-two Navy and Marine Corps air stations, including principal stations in the United States, and islands of the Caribbean Sea and the Pacific Ocean, the in-place subgrade moisture has been found to exceed only occasionally the optimum by as much as 3% for fields that have been in use for from four years to nine years.[7] Many of these field studies were made during wet seasons and at the time of spring thaw.

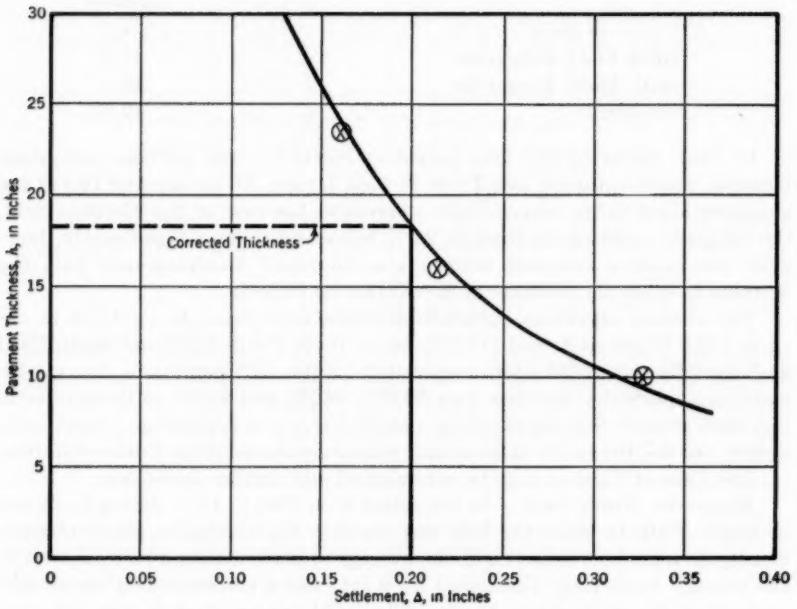


FIG. 5.—PLOTTING OF DATA OBTAINED BY LOADING THE TRIAL SECTIONS

The plotting of data obtained by loading on trial sections is shown in Fig. 5. In each of the three panels of the trial pavement section (Fig. 3) the pavement thickness is corrected as may be indicated for possible future increase of subgrade moisture in the manner already described. The corrected thicknesses are then plotted against the observed settlements and the best possible curve is drawn through the points. The thickness corresponding to 0.2 in. of plate settlement is taken as the over-all design pavement thickness. In constructing the trial sections, the asphaltic surfacing is omitted—an equivalent thickness of base course being substituted for it.

[7] "Pavement Evaluation by Loading Tests at Naval and Marine Corps Air Stations," by L. A. Palmer and James B. Thompson, *Proceedings*, Second International Conference on Soil Mechanics, Vol. 2, 1948, p. 227, Table 1.

Pavement design and evaluation on the basis of plate loading tests have been found to be generally reliable and economical procedures in Navy experience. It is hoped that the field tests may be simplified and reduced by finding correlations between laboratory test data obtainable from soil tests such as unconfined compression tests or in-place CBR tests (or both) and the E_r moduli as determined by plate loading.

Trial sections for concrete pavement involve only the construction of compacted subbase panels 20 ft square in cut-and-fill areas of runways, taxiways,

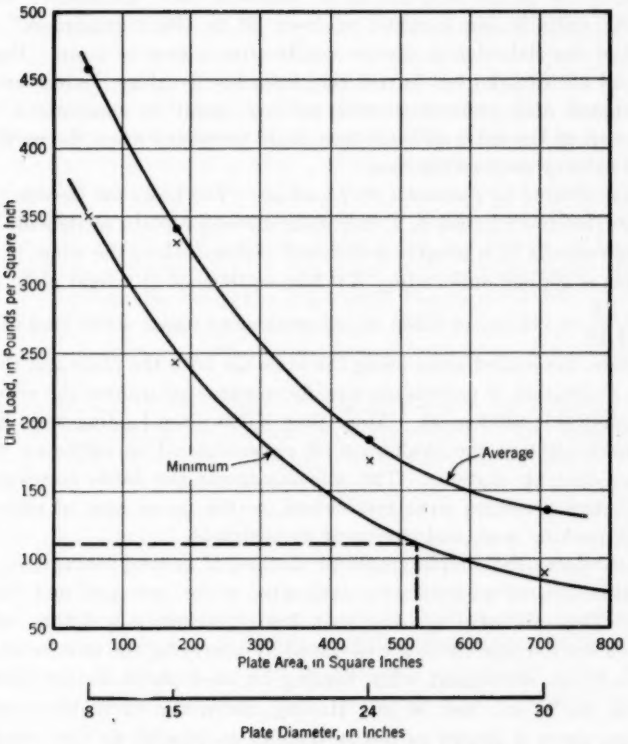


FIG. 6.—EXAMPLE OF THE GRAPHICAL METHOD OF DETERMINING THE SAFE WHEEL LOADS FOR FLEXIBLE PAVEMENTS

aprons, and runway ends, and at transitions between cut-and-fill areas. In the use of the well-known Westergaard analysis, the design requirement is that when loading on the compacted subbase, the foundation modulus, k, must not be less than 200 lb per cu in. after a reasonable correction (based largely on experience) has been made for probable future increase in moisture of the compacted subgrade and subbase. The policy requires use of a factor of safety of about 1.4 based on values of the moduli of rupture obtained by testing beams

after 28 days of wet-curing. Extensive pavement evaluation studies have shown that such a factor of safety at 28 days may reasonably be expected to increase to about 1.6 at the end of a year in the actual pavement, and the elapse of a year may be expected between the time of construction and the time when the pavement begins to carry a considerable volume of traffic of the heaviest type planes. Subbase thicknesses have varied from 6 in. to 18 in. as determined by loading on trial sections with a plate 30 in. in diameter in all cases. The primary consideration is to obtain a permanency and uniformity of k-modulus throughout the paved area during its period of use. Experience has proved that with initially low k-values of from 50 to 100, "mushroom" subgrade support of the slab almost always results after a year or more. Pumping at joints and all attendant evils resulting from low k-values render the cost of a well-designed and well-constructed subbase small in comparison with the greater cost of the extra maintenance made necessary when the earth support is given little or no consideration.

Design Proved by Pavement Performance.—The basis for flexible pavement design as described herein is a maximum allowable plate settlement of 0.2 in. This corresponds to a greater settlement realized when the same load on the same area is applied uniformly. A 0.2-in. settlement of a rigid plate is equivalent to $\frac{1.5}{1.18} \times 0.2$ in., or 0.254 in., of settlement under a tire load of the same magnitude, the loaded areas being the same for both the plate and the tire.

The evaluation of pavements already constructed utilizes the criterion of a 0.2-in. allowable settlement. More than 3,000 plate loading tests have been made since 1944 in the evaluation of pavements at seventy-two Navy and Marine Corps air stations. The safe maximum tire loads corresponding to different tire pressures were established for the pavements of each runway, taxiway, parking area, and similar at each airfield.

Fig. 6 shows the simple graphical method of accomplishing this. In this illustration, two curves are shown, designated as the "average" and "minimum" curves. The ordinates are the unit loads producing a 0.2-in. settlement. Points on the average curve are obtained by averaging the unit loads that produced a 0.2-in. settlement when loading on steel plates having diameters of 8 in., 15 in., 24 in., and 30 in. Having drawn the curve for averages, the minimum curve is drawn as nearly parallel as possible to the average curve and through the points of minimum values obtained by loading the flexible runway pavement at different carefully selected sites. The minimum curve is used for evaluating the pavement in terms of tire-load capacity. The tire contact pressure on the pavement is taken as the tire pressure increased by 10%. Thus, for a tire pressure of 100 lb per sq in. (Fig. 6) the contact pressure is 110 lb per sq in., and this ordinate value on the minimum curve corresponds to a plate area (also a tire contact area) of 525 sq in. and a safe maximum tire load at this tire pressure of $525 \times 110 = 57{,}750$ lb, or 58.0 kips to the nearest half-kip. Additional results thus obtained are tabulated as follows:

Tire pressure, in pounds per square inch	Contact pressure, in pounds per square inch	Safe wheel load, in kips
80	88	58
100	110	56.5
140	154	55
180	198	53
220	242	46
260	286	37
300	330	23

Many of the pavements evaluated by this procedure were originally designed by the method of trial pavement sections. Because of the progressive compaction of base and subgrade materials by traffic, the general tendency was toward an increase in load capacity, subsequent to construction, beyond that for which the pavement was originally designed. This tendency was especially marked in the case of sandy subgrades. Most of the pavements were designed and constructed during World War II and were intended to accommodate only light training planes. In these cases the pavement thicknesses were determined by judgment and not by trial sections or any other test method of pavement design. After two or more years of use, most of these pavements withstood the traffic of planes having single tire loads of 30 kips or more without damage beyond abrasion of the thin asphalt surfacing. This was quickly remedied by resurfacing. A description of the pavements of twenty of the airfields was published in 1946.[8]

With the exception of one runway (Naval Air Station, Los Alamitos, Calif.) the writer knows no instance of pavement failure caused by tire loads that did not exceed the rated capacities. The error at Los Alamitos resulted from too few loading tests. This was subsequently corrected. At a considerable number of the airfields the tire loads exceeded the rated capacities by from 20% to 50% without noticeable damage. At the Naval Air Station, at North Kingsville, Tex., tire loads of from 30 kips to 37 kips, at tire pressures of approximately 90 lb per sq in., caused complete failure in certain sections of pavement rated at 12.5 kips and 15 kips at 90 lb per sq in. In other sections of the same two runways, no damage was noticeable. Failure was mostly at the runway ends where there was dynamic loading on take-offs. At the Naval Air Station, Johnsville, Pa., there was repeated failure under tire loads exceeding the rated load capacity by 50%.

The basic principle utilized in the evaluation of the load capacity of a pavement is the same as that which is applied in the method of loading trial pavement sections when designing new pavements. The 0.2-in. allowable plate settlement has proved to be a reliable index for both new design and evaluation purposes. The Burmister theory (which is thoroughly sound from every angle) is not the yardstick of design. It is a means of obtaining a first estimate of the required pavement thickness for the construction of trial sections of different thicknesses. There must be no misinterpretation of the procedure from this

[8] "The Evaluation of Wheel Load Bearing Capacities of Flexible Types of Pavement," by L. A. Palmer, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., Vol. 26, 1946, pp. 72–97.

standpoint. The procedure used by the Bureau of Yards and Docks is a mixture of the rational and empirical methods.

The pavement evaluation data were used in estimating the required extent of "beefing up" of the old pavements for aircraft considerably heavier than the rated tire-load capacities.

The Reinforcement of Old Pavements

Flexible-Type Pavement.—Values obtained from the plate loading tests made on the subgrade and on the surface of the existing pavement during the course of extended field studies, when the pavements at the air stations were evaluated, made it possible to estimate (by means of theory) the additional overlay in the form of flexible pavement reinforcement that was required to sustain safely tire loads that would exceed the rated capacity of the existing pavement. In this connection, there is no more reason to distrust theory as related to flexible pavement than to distrust the theory that has been applied for years in the design of concrete pavement—or to distrust the use of handbooks, all based on various theories, that engineers use generally.

Typical Example.—A runway at the Naval Air Station at Memphis, Tenn., is to be reinforced to carry a 50-kip load on a single tire at a tire pressure of 150 lb per sq in. The measured over-all pavement thickness is 8 in., of which 6 in. are base course and 2 in. are asphalt surfacing. At this field 1 in. of the surfacing is equivalent to 2 in. of base in load-bearing capacity. Thus, in computations, the pavement is considered as a single layer—as base course having a thickness of 10 in. By loading on the subgrade with the plate of 30-in. diameter, a value of 25 lb. per sq in. is conservative for producing a settlement of 0.2 in. By loading with the same plate on the surface of the pavement, consideration of all the data shows that a conservative value of 58 lb per sq in. corresponds to 0.2 in. of plate deflection. The subgrade moisture was at about 2% above optimum in this runway after it was used six years. It is not assumed that it will become wetter after reinforcement of the old pavement.

Then, E_2 from Eq. 1 is 2,213 lb per sq in. and F, from Eq. 2 (with $w = 58$), is 0.41. By interpolating in Fig. 4 for $F = 0.41$ and $h = 10$ in. $= 0.67\ r$, the ratio $\frac{E_2}{E_1}$ is $\frac{1}{50}$. Then $E_1 = 110{,}650$ lb per sq in. Use Eq. 3, setting $r = 10$ in. and $w = 165$ lb per sq in., corresponding to a uniformly distributed load of 165 lb per sq in. on a circular area having a radius of 10 in. (a total load of 51,810 lb, which is close enough to 50,000 lb for estimating). From Eq. 3, the new F-value is 0.18. Interpolating again in Fig. 4 to find a value of $\frac{h}{r}$ corresponding to $\frac{E_2}{E_1} = \frac{1}{50}$ and $F = 0.18$, it is seen that $\frac{h}{r} = 1.7$, so that $h = 1.7 \times 10 = 17$ in., which is the total pavement thickness required. Therefore, the required additional overlay must be equivalent to 7 in. of material having an E_1 of 110,650 lb per sq in.; that is, the reinforcement must produce a spreading of the load equivalent to 7 in. of the existing, well compacted base course. This requirement must be checked by plate loading tests.

Judicious use of loading tests as a yardstick in pavement design must necessarily be based on long study and experience. Usually, such experience is not amenable to analysis and cannot be passed on to others in written form. Various contradictions may arise when the loading tests are not preceded by thorough analyses of soils, pavement materials, and topographic and drainage studies of the site. The in-place densities and moisture contents of all materials must be the same if direct comparisons of the relation of different pavement thicknesses to the plate settlement are to be made. Failure to eliminate all variables other than pavement thickness invariably leads to erratic and unreliable results.

In the foregoing example, it must be considered that the additional 7 in. of pavement reinforcement must be firmly bonded to the old pavement. Cleaning and tack coating of the old pavement before applying reinforcement are

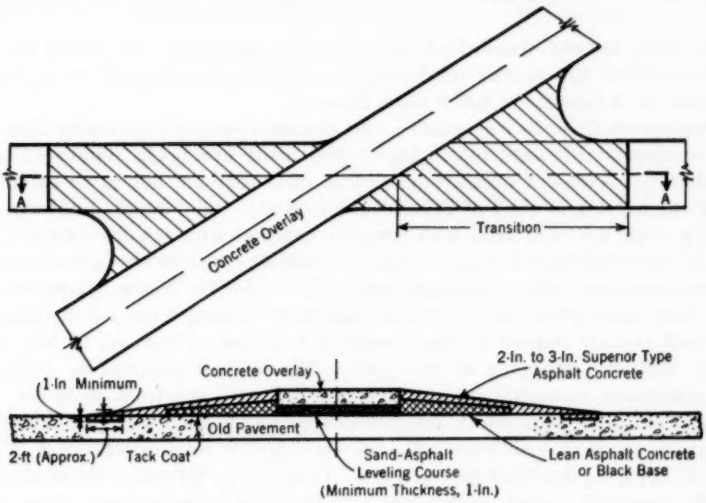


FIG. 7.—GENERAL SCHEME FOR RAMPS AT RUNWAY INTERSECTIONS

essentials. Overlays that slide on the old surface must be avoided. Base courses of the bituminous types, such as black base and penetration macadam, have distinct advantages in this connection. Flexible-type pavements are adaptable to stage construction, but the additional reinforcement must become an integral part of the entire built-up pavement.

In very few instances in the 1950–1953 rebuilding program of the Bureau of Yards and Docks have old pavements been dug out, spoiled, and replaced with new materials, although some engineers have recommended this extravagant procedure. The old pavement is an investment and should be utilized. Evaluation studies have proved conclusively that (at least at naval air stations) old pavements usually have been pounded down by traffic to higher densities and bearing values than they had when newly constructed. Furthermore, from the standpoint of effective surface drainage, it is invariably advantageous to keep

the pavement high in relation to the surrounding terrain. In one instance out of forty it was found advisable to scarify an old, poorly designed, and poorly constructed pavement and recompact it to produce a higher in-place CBR and a greater density before adding the reinforcement overlay.

If the reinforced runway pavement intersects a taxiway or runway that is not to be reinforced, the problem of transitions is encountered. In the present practice (1953) of the Bureau of Yards and Docks, the built-up runway pavement is "ramped" down on the adjacent paved areas with asphaltic concrete of quality equal to that used in surfacing landing strips for jet airplanes. Fig. 7 shows the usual scheme that is followed. At the outer end of each transition, grooves are sawed in the concrete and a keyway is chipped out. The length of the transition is determined by the difference in elevation between the old pavement and the overlay; the transition is ramped at a rate of change of grade not to exceed 0.3% per 100 ft. The longitudinal slope is not allowed to exceed 1%.

At those runway ends which are of flexible pavement and which are not to be extended, the overlay reinforcement of the end portions is of reinforced concrete for a distance of 500 ft from the end.

Overlays on Concrete Pavements.—Old concrete runway pavements that will be overstressed by anticipated heavy tire loads are being reinforced with flexible-type pavement overlays, except at ends that are not to be extended, where the overlay in the last 500 feet is concrete. Old concrete taxiways and parking areas are reinforced with concrete overlays with few exceptions.

The preferred flexible-type overlays on existing concrete are asphaltic penetration macadam, with a maximum size of 2.5 in. for the coarse aggregate, and black base made with densely graded aggregate ranging from 1.5 in. down to fines and asphalt cement having penetration grades of from 85 to 100, with a 3-in. asphaltic surfacing in each case. The asphalt penetration grade for the penetration macadam to cover concrete is kept low—from 40 to 60. All old concrete pavements are thoroughly cleaned and primed with not more than 0.1 gal per sq yd of rapid-cure cutbacks, emulsion, or road tar (RT-2 or RT-3) prior to applying flexible-type overlays. Leveling, to fill minor depressions, is usually done with a sand-asphalt mix. In this case it is expedient, economical, and generally adequate to rake out dirt-filled and poorly-bonded seals in the old concrete joints and thereafter fill them with the sand-asphalt mix prior to constructing the flexible over-lay. Sometimes the only overlay required is the asphaltic concrete surfacing.

In estimating the thickness of required flexible overlay on concrete pavement, the Burmister analysis for the three-layer system is followed. In this procedure, the modulus E_3 for the subgrade is computed from the k-modulus determined from previous evaluation studies, and E_2 for the concrete is assumed to be 4,000,000 lb per sq in. It is safe to assume E_1 for the bituminous overlay. A value of 100,000 lb per sq in. is conservative for penetration macadam or black base that is always covered with 3 in. of a superior type of asphaltic concrete surfacing. In terms of k, obtained by loading the earth under the concrete,

$$E_3 = 1.18\, r\, k\, F \qquad (4)$$

The allowable settlement, Δ(under the tire load in this case), is taken as 0.1 in. Greater deflections would involve cracking of the concrete. However, the allowable settlement in this case should be determined with reference to the actual deflection of the concrete slab, exclusive of the "plastico-viscous" yielding within the bituminous overlay.

When the subgrade support under concrete is firm to the extent that overloading of a "cracked out" piece of the slab does not push the piece downward into the earth, thus forming a local depression, it is observed that the asphalt surfacing covering an old concrete pavement tends to remain intact without either noticeable cracks or unevenness. This favorable condition may even continue when there is progressive cracking of the concrete through repeated overloading. The cost of maintenance of joints in old concrete is eliminated by a well-designed and well-constructed bituminous overlay, although other maintenance costs, such as periodic seal coating, are made necessary.

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